

APPENDIX A  
ENGINEERING

Ponce de Leon Inlet  
Feasibility Study  
Engineering Appendix

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# **ENGINEERING APPENDIX**

## **INTRODUCTION**

This appendix describes the procedures used to investigate alternative plans of improvement for the Ponce de Leon Inlet Feasibility Study. This appendix describes the physical and numerical modeling efforts, and contains sections on the detailed design of the recommended plan, geotechnical data, and final cost estimates. The selected plan consists of a 1000-foot seaward extension of the south jetty.

Construction of a 2340-foot revetment extending westward from the north jetty will be performed with Operations and Maintenance (O&M) funding beginning in FY1999, and will be assumed to be in place for the with and without project conditions. Due to its importance on the inlet system, discussions of the revetment design, location, and dimensions will also be presented in this appendix.

## **PHYSICAL AND NUMERICAL MODELS**

### **GENERAL INFORMATION**

Physical and numerical models were used to examine the effects of various proposed alternatives on the inlet hydrodynamics, with the ultimate goals of improving safety and reducing maintenance at Ponce de Leon Inlet. The physical modeling effort was conducted by the U.S. Army Corps of Engineers' Waterways Experiment Station, Coastal Hydraulics Laboratory (CHL), formerly the Coastal Engineering Research Center-CERC. The numerical modeling was conducted by Taylor Engineering, Inc, located in Jacksonville, Florida. Taylor Engineering, Inc. is the engineering consultant for the local sponsor, the Ponce de Leon Inlet Port Authority. This section will provide a brief summary of the physical and numerical modeling efforts. Detailed information on both modeling efforts is provided in the "Supplemental Report: Ponce de Leon Inlet Feasibility Study, Physical and Numerical Modeling Studies".

The modeling efforts conducted by CHL and Taylor Engineering, Inc. were very closely coordinated. Both models were created using the same hydrographic surveys and field data, which are described below. Information and data were continuously exchanged between CHL, Taylor Engineering, and the Jacksonville District during model development and production. Numerous meetings, site visits, and teleconferences were held throughout the duration of the studies to ensure that model setup, calibration, verification, and operations proceeded in a mutually compatible manner.

The same alternative plans of improvements were evaluated using both models. Specifically, these plans of improvement were:

1) Seaward extension of the south jetty - The purpose of extending the south jetty is to reduce the northward transport of sediment around the seaward end of the south jetty into the inlet, and to more evenly distribute tidal flow across the inlet. By reducing the transport of sediment into the inlet, the large shoal in the inlet throat would gradually erode, and the navigation channel would tend to migrate southward toward the center of the inlet. Two alignments were examined for the south jetty extension: straight (aligned 20 degrees north of east) and parallel to the north jetty (aligned due east). Extension lengths of 500, 800, and 1000 feet were examined for each alignment (500 and 1000-foot extensions were examined in the numerical model). The 1000-foot alternative would extend the south jetty tip to the same easterly limit as the north jetty tip. The jetty crest elevation was optimized at +7.0 feet mhw for the original design; this crest elevation would be maintained along the 1000-foot jetty extension.

2) Re-opening the weir in the north jetty - The purpose of re-opening the weir in the north jetty is to provide a second outlet for tidal flow through the inlet and to allow limited transport of sediment into the inlet. By allowing a portion of the tidal flow to pass through the weir, tidal current velocities and associated scour through the outer portion of the inlet throat would be reduced, providing safer conditions for boaters. Allowing limited sediment transport into the north side of the inlet could protect the foundation of the north jetty from the tidal current scouring which is currently threatening the landward portion of the jetty. Accumulation of sediment along the weir portion of the jetty would tend to displace the navigation channel southward, away from the north jetty and more toward the center of the inlet. Weir openings of 500, 1000, and 1500 feet were evaluated. The 1500-foot opening duplicated the original weir configuration, and the 500 and 1000-foot weirs each extended landward from the position of the eastern end of the original weir. All modeled weir crest elevations were 0.0 feet, mhw.

3) Engineered channel through sand spit - A channel would be constructed through the sand spit inside the inlet (south of the landward end of the north jetty) in order to relieve erosional pressure on the north jetty created by the high ebb flow velocities along the jetty. Construction of this channel would reduce the volume of flow through the dogleg portion of the channel, which is currently located south of the sand spit. Tidal flow would be more aligned with the axis of the channel, and would be more uniformly distributed across the inlet cross-section. A revetment is already planned for construction along the shoreline on the north side of the channel alignment in order to prevent further migration of the shoreline to the north, which could result in flanking of the north jetty and damage and disruption to several marinas located immediately north of the proposed channel. This revetment is to be constructed beginning in FY 1999 using Operation and Maintenance (O&M) funding.

The original scope of work required that each of the above alternatives be simulated in both models, in order to determine the effects of the proposed improvements relative to the existing conditions. Existing conditions included the construction of the revetment extending westward from the landward end of the north jetty. The shoreline inside the inlet was not allowed to recede northward past the revetment alignment.

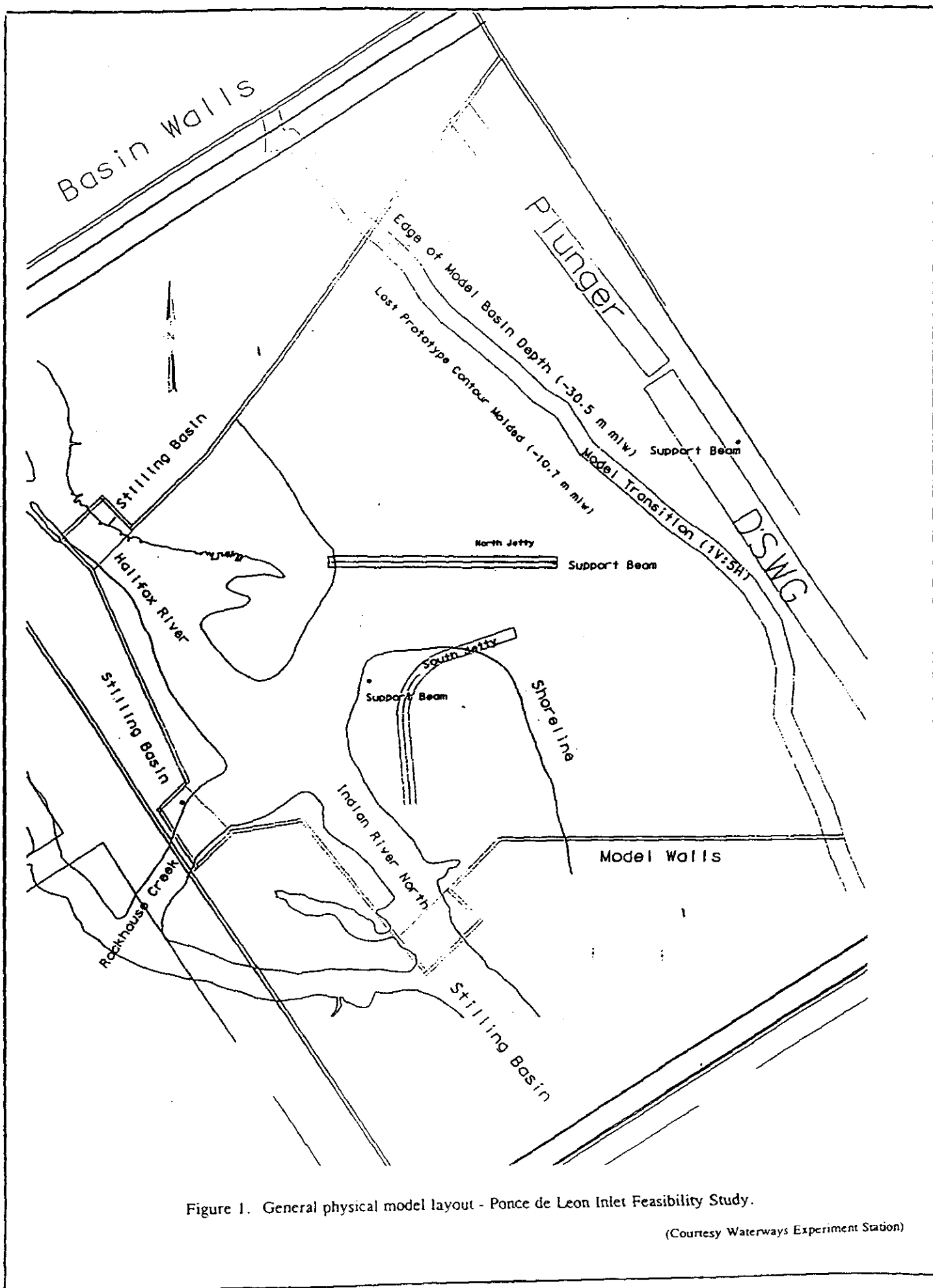
The physical model was a 3-dimensional steady-state model which simulated either peak ebb or peak flood conditions, and could create a variety of storm wave events. This model was used to examine wave and current-driven sediment transport using gage measurements and dye and tracer studies. In this manner, numerous tidal current/storm wave interaction scenarios were modeled, and the effects of adding each of the proposed plans of improvements (or combinations of improvements) were studied.

The numerical hydrodynamic model examined the effects of the alternative plans of improvement on inlet tidal hydrodynamics. Patterns of erosion and shoaling for each alternative were inferred through the use of sediment transport analyses. The setup, operation, and results of each modeling effort will be discussed in the following sections. These sections present an overview only, and detailed information on all aspects of both modeling efforts are contained in the supplemental report "Supplemental Report: Ponce de Leon Inlet Feasibility Study, Physical and Numerical Modeling Studies".

## DESCRIPTION OF PHYSICAL MODEL

Work performed by CHL consisted of field measurements and construction and operation of the physical model. The field data gathered by CHL included a 2-month record of water level measurements at four locations inside the inlet and two locations in the ocean, north and south of the inlet. Current velocities were measured along eight transects in the inlet throat and north and south access channels using a vessel-mounted current profiler. Non-synoptic current velocities were measured during a 13 hour period at each transect, in order to cover the peak ebb and flood velocities. Current velocities were also measured in the inlet throat and the north and south access channels using bottom-mounted current meters. These hydrodynamic data were used in conjunction with hydrographic surveys obtained by CHL (LIDAR surveys) and the Jacksonville District (conventional surveys) in July/August 1994 to construct and calibrate the physical model.

The physical model is an undistorted three-dimensional steady state model, which was constructed at a 1:100 scale in one of CHL's physical modeling facilities at the Waterways Experiment Station in Vicksburg, Mississippi. The model includes the inlet throat area, and extends approximately 1 mile north and south along the access channels, and into Rockhouse Creek. The model accurately reproduces offshore bathymetry to the 35-foot depth contour. The layout of the model is shown in figure 1. As shown in figure 1, two wave generators (plunger: non-directional and DSWG: directional) were used to create the north and south-directed wave fields used in this study. A water circulation system was used to generate and maintain the peak ebb and flood tidal flows in the model. Field data or numerical model data were used to determine the water surface



elevations corresponding to peak flow. These water surface elevations were then duplicated and held constant in the physical model for ebb and flood flow simulations.

The physical model was equipped with 13 wave gages and 5 velocity meters. These instruments were placed throughout the model to determine wave heights and current velocities under a given set of wave/tidal current/tidal elevation conditions, for comparison to similar measurements taken with alternative plans of improvement in place. Both wave generators, the water circulation system, and all gages were computer-controlled. All gage data were recorded on a personal computer, and were capable of being viewed in real-time. The physical model was validated by comparison of model water level and velocity measurements with the field data.

Wave conditions used in the physical model were derived from the 40-year Wave Information Study (WIS) hindcast records for WIS stations 21 and 22 (south and north of the inlet, respectively). Ten wave conditions were ultimately selected for use in the model. Four of these conditions were "safe" navigation conditions, as defined in the supplemental report as waves of less than 1.5 meter in height, which can be safely negotiated by boats 25 feet long or greater. The other 6 cases were storm waves which were capable of transporting the largest volumes of sediment into the inlet. These 6 storm wave cases were chosen to evaluate the effectiveness of the various south jetty extensions (waves from the south). Likewise, a different set of storm wave conditions was developed for investigation of the north jetty weir opening option (waves from the north). The "safe" navigation wave conditions referred to herein as "normal" waves developed for the south jetty extension were repeated for the weir opening alternatives.

Sea level rise has been determined using a variety of methods along central Florida's Atlantic coastline. From analyses performed in the St. Johns County General Reevaluation Report dated January 1998, changes in sea level were calculated along the St. Johns County shoreline, located about 50 miles north of Ponce de Leon Inlet. These rates of sea level rise are applicable to the Ponce de Leon Inlet study area. Using the National Research Council method, sea level rise rates of 0.85 ft, 1.34 m, and 1.87 ft were estimated by the year 2050 for the project area, based on the low, medium, and high coefficients of sea level change, respectively. The low rate corresponds closely with a calculated rise of 0.75 ft by the year 2050 using the EPA method. Using the shoreline recession method developed by Per Bruun, the calculated rise in sea level is 0.8 feet by the year 2050, corresponding to a shoreline recession of 0.8 feet per year. The 1% chance of exceedance estimate of sea level rise is 1.8 feet by 2050, corresponding to a shoreline recession of 1.7 feet per year. Using the shoreline erosion method, the most likely recession rate of 0.8 feet per year translates into a loss of 0.9 cubic yards per linear foot of shoreline per year.

#### DESCRIPTION OF NUMERICAL MODEL

The numerical hydrodynamic model used by Taylor Engineering, Inc was the TRANQUAL model, developed by Taylor and Dean in 1972, and updated and refined by

Taylor and Pagenkopf in 1981. TRANQUAL is a vertically integrated two-dimensional model which uses an implicit finite difference methodology to solve the equations for conservation of mass and fluid dynamics. The model simulates time varying water level elevations and ebb and flood tidal current velocities over selected tidal cycles. The model includes the effects of nonlinear propagation of long waves in shallow estuaries, and includes the effects of irregular basin geometry, flooding and drying of intertidal areas, bottom friction, wind shear, freshwater inflows, convective and Coriolis accelerations, gravity, lateral shear, and small-scale features such as flow constrictions and sills.

The same July/August 1994 survey of Ponce de Leon Inlet which was used to construct the physical model was used to set up the numerical model. The area of coverage for the numerical model is larger than the area covered by the physical model. The model boundaries are shown in figure 2, and cover an area of 2.7 by 4.3 miles. This area was gridded, with a grid size of 100 by 100 feet, arranged in a 140 by 225 matrix. A three-dimensional surface was generated using digital terrain modeling (DTM) software, and an elevation value was assigned to each grid cell using this surface.

The TRANQUAL model allows for four types of boundary conditions: zero flow (land, impermeable barriers, etc), time varying (tide) or constant water surface elevation, time varying or constant flow rate, and free surface profile radiation. The specific boundary conditions selected for the Ponce de Leon Inlet numerical modeling study area were: a specified ocean tide linearly interpolated between the two eastern corners of the model, a free surface profile radiation applied uniformly across all offshore grid cells along the northern and southern boundaries, a specified uniform tide (Fourier series representation) across the open water boundary cells in the Halifax River, a free surface profile radiation boundary condition uniformly applied across the boundary cells of the Indian River North, Spruce Creek, Redland Canal, and Smyrna Creek. No flow was permitted at all other boundary elements (which are dry land), no flow was permitted across the north or south jetties, and no freshwater inflow or wind effects were included. Data collected by CHL from July-August 1994 (as described above) were used to set up and calibrate the TRANQUAL model boundary conditions.

The model's wetting and drying algorithm which allowed the simulation of the flooding and drying of marsh and shoals was an important feature during model validation. The model was validated with comparisons between model generated water level and velocity conditions and the field data. The two tidal flow scenarios which were numerically modeled were the 21-22 July 1994 spring tidal cycles, and a 1-year storm tide superimposed on these cycles. A 30-day lunar month cycle was also examined.

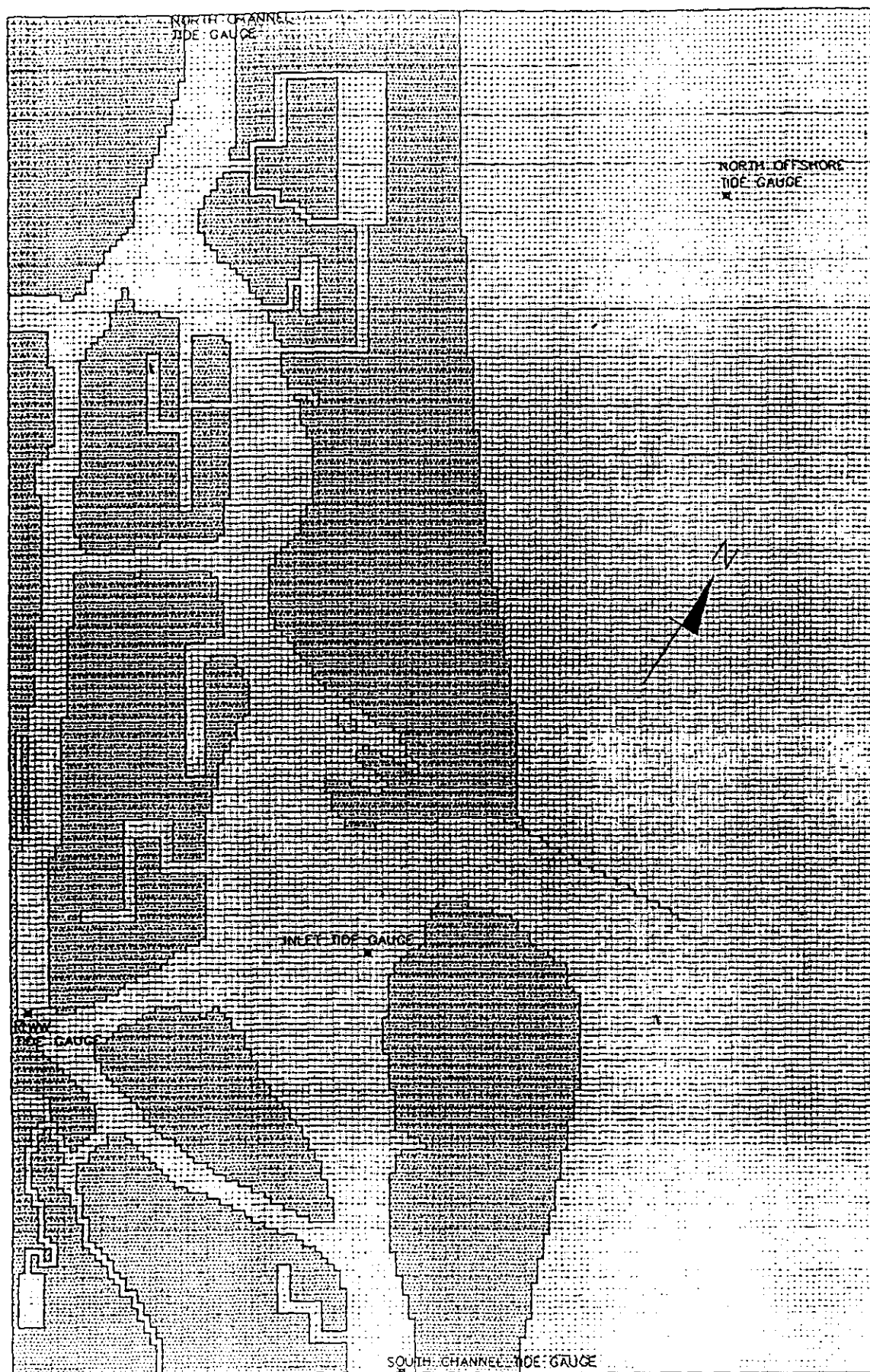


Figure 2. Numerical model grid scheme - Ponce de Leon Inlet Feasibility Study.

(Courtesy Taylor Engineering, Inc.)

## VOLUMETRIC ANALYSIS

In addition to the numerical modeling effort, a volumetric analysis was conducted by Taylor Engineering, Inc. in order to quantify regions of erosion and accretion. Only the 1986, 1990, and 1994 surveys were used in this analysis since they are representative of the inlet in its present configuration, following closure of the north jetty weir in 1984. Three periods of analysis were examined: 1986-1990, 1990-1994, and 1986-1994.

The most dominant trends observed in the volumetric analysis were the erosion of the sand spit inside the inlet, and erosion along the landward end of the south side of the north jetty. The erosion of this spit is primarily due to the flood tidal flow impinging on the area, and the erosion along the north jetty is due to the ebb tidal flow impinging on the north jetty. Both erosional areas result from the deflection of tidal flow caused by the dogleg configuration of the channel around the sand spit. As the sand spit continues to erode, the dogleg becomes straighter and erosional pressures are expected to ease. The flood shoal area and the shoal in the inlet throat were generally accretionary during this period. Bathymetric change charts and a detailed discussion of volumetric changes in the inlet are presented in the supplemental report. Gross volumetric changes are 243,000 cy/yr between 1986-94. The net channel shoaling rate, based on dredging records, is 51,000 cy/year from 1952-96, and 46,000 cy/yr since closure of the north jetty weir in 1984.

## PHYSICAL AND NUMERICAL MODELLING OF ALTERNATIVE PLANS

Extension of South Jetty. The first alternative plan of improvement to be physically and numerically modeled was the seaward extension of the south jetty. Previous studies had shown that construction of a south jetty extension would reduce the transport of sediment into the inlet, reducing shoaling in the inlet throat and improving navigation by allowing the channel to migrate southward toward the center of the inlet. Two alignments were examined using both models: a straight extension of the existing south jetty alignment (20 degrees north of east), and an alignment parallel to the north jetty (due east). The physical model examined both alignments with 500, 800 and 1000-foot extensions; the numerical model examined both alignments with 500 and 1000-foot extensions. Model results were used to select the optimum south jetty extension length and alignment.

Dye tests in the physical model quickly indicated that the shorter jetty extension lengths of 500 and 800 feet (for both alignments) were not effective in preventing the potential for bypassing of large volumes of sediment. Jetty extension lengths of 1000 feet were effective in preventing dye and tracer movement into the inlet for most wave/current scenarios, and only minimal dye movement into the inlet was observed using both alignments (at 1000-foot length) for the "worst case" storm condition. Likewise, the inferred sediment movement based on tidal current velocities in the numerical model indicates that the longer jetty length should allow less sediment to enter the inlet. The 1000-foot extension also produced more desirable hydraulic effects, i.e. a more even distribution of flow across the inlet cross-section. These model results verified the

intuitive assumption that the longer jetty length should be more effective in preventing sediment transport into the inlet.

Both models examined the effects of the two alternative jetty extension alignments. The physical model was inconclusive; dye response was similar for both alignments. The numerical model produced more definitive results. Both straight extensions (500' and 1000') produced generally undesirable hydraulic effects in the numerical model, such as reduced velocities in the middle of the entrance channel and increased velocities in the vicinity of the scour hole at the landward end of the north jetty. The alignment parallel to the north jetty provided a more even distribution of tidal flow across the inlet cross-section, pulling the channel away from the north jetty slightly. Due to the more desirable flow characteristics of the wider parallel alignment, the 1000-foot parallel extension of the south jetty was chosen as a recommended plan. The numerical model results indicated that this alternative only influenced hydrodynamic conditions east of the proposed location of the north jetty weir and had no influence on interior hydrodynamics or associated sedimentation.

Three nearshore gages were positioned in the physical model to determine the effects of the south jetty extension on wave heights at the popular surfing area immediately south of the south jetty. Gage data showed an average of 10 percent wave height increase during normal ebb flows, and no discernible wave height changes during flood flows. This data assumes no change in offshore bathymetry from the 1994 survey; extension of the jetty would probably cause significant accretion in this area, which could result in a southward and eastward (offshore) displacement of the present surfing area.

North Jetty Weir Opening. The next alternative to be evaluated in the physical and numerical models was the re-opening of the weir in the north jetty. The original weir was built during the initial construction of the north jetty in 1969, and was closed in 1984 due to excessive shoaling in the inlet due to the inflow of sediment from the north beaches through the weir. Re-opening the weir would allow a portion of the tidal flow to pass through the weir instead of between the jetties, reducing current velocities and associated scouring, and increasing boating safety at the offshore entrance to the inlet. Re-opening of various lengths of weir is also being investigated in order to allow limited volumes of sediment to enter the inlet. The limited inflow of sediment through the weir could alleviate the scouring problem at the landward end of the north jetty, and partially rebuild the shoal in this area, forcing the channel southward toward the center of the inlet.

Possible negative effects of re-opening the weir include re-establishing strong cross-currents across the weir, which presented a serious hazard to boaters when the original weir was in place. Increased wave energy through the weir (broadside) presented another hazard to boaters. Finally, tidal velocities were too strong to allow deposition in the former impoundment basin area. Sediment which was transported through the original weir into the impoundment basin was quickly transported to the sand spit, and ebb and flood shoal areas. Under the present inlet configuration, it was hoped that a limited weir opening might sufficiently reduce tidal flow through the inlet entrance

channel and allow enough sediment into the inlet to alleviate the scouring, without adversely impacting navigation.

The three weir lengths used in both models were 500, 1000, and 1500 feet. All three weirs would extend landward from the seaward end of the original weir. This point of origin was selected in order to keep the weir roughly centered on the outer portion of the littoral zone north of the jetty. The 1000 and 1500-foot options would each involve removal of a portion of the concrete fishing walkway, which was constructed by the local sponsor following weir closure. Due to the demonstrated positive effects of constructing the south jetty extension, both models assumed that the 1000-foot south jetty parallel extension was in place in combination with each of the weir openings.

The physical model used dye studies to trace the movement of peak tidal flows through the weir under a variety of scenarios of differing wave conditions, current velocities, and water levels. Results from the physical model showed that on the ebb flows most of the discharge flowed out through the inlet mouth (between the jetties) through the existing deepwater channel along the north jetty. Flow over the weir was minimal. During flood flows most of the water enters the inlet through the mouth, and flow through the weir is also minimal. Waves from the north tended to greatly increase the flow of water over the weir. Numerical modeling performed by Taylor Engineering, Inc verified that for the design sill elevation of 0.0 feet mhw, a very small percentage of tidal flow passes over the weir. More importantly, both models showed that regardless of the length of weir opening, the ebb flow velocities along the north jetty were minimally decreased, even at storm water levels. This suggests that the tidal scouring of the base of the north jetty would not be reduced by re-opening a weir in the north jetty. This also suggests that substantial accretion of sediment transported through the weir is unlikely, due to the excessive current velocities. Since the closure of the weir, depths in the vicinity of the original weir along the south side of the jetty have increased from 10 - 25 feet to 40 - 50 feet. This increased depth allows a high percentage of flow to avoid the weir influence and exit the inlet mouth.

Tracer (coal dust) was used to determine the relative transport rates of sediment through the various weir opening lengths in the physical model. A line of tracer was placed adjacent to each weir opening along the north side of the jetty and various storm wave scenarios were simulated. For each of the weir openings, material was transported through the weir and deposited in the scour hole area, but tidal currents tended to quickly transport the tracer material out of the scour hole. During normal peak ebb flows and navigable waves, almost no sediment was transported into the inlet. By increasing the wave heights and periods, some sediment was deposited in the scour hole for each weir opening. Under flood conditions and normal waves, more material was observed crossing the weir into the inlet, with roughly the same pattern of deposition in and around the scour hole. When storm waves were added however, virtually all tracer material was transported through the weir and into the inlet. For peak ebb normal flows tracer was deposited in the scour hole, but for peak flood normal flows tracer was deposited up to 400 feet south of the north jetty, and very little was deposited in the scour hole. These

physical model tracer studies indicated that re-establishment of an impoundment basin in this area is unlikely due to the strong tidal currents.

Numerical modeling by Taylor Engineering, Inc. verifies that re-opening the weir in the north jetty produces minimal positive impacts on the hydraulic processes of Ponce de Leon Inlet. Model simulations indicate that opening the weir will cause a slight decrease in inlet tidal velocities east of the weir, particularly during flood tides. This, in combination with increased inflow of sediment through the weir, would probably increase shoaling, resulting in higher channel maintenance. Model runs on the 1500-foot weir opening showed that increased tidal velocities through the weir could cause beach erosion north of the jetty. Finally, the numerical model shows that re-opening the weir would not provide the desired hydrodynamic and sedimentation modifications intended. Based on the results from the physical and numerical models, re-opening the weir is not recommended.

Constructed Channel Through the North Spit. The third alternative plan of improvement consists of excavating a channel through the sand spit. This alternative was proposed in order to straighten the landward portion of the entrance channel (removing the dogleg), reducing erosional pressure on the north jetty. Ebb tidal flow would no longer impinge on the north jetty, and tidal flow would be more evenly distributed across the entrance. A revetment would be constructed along an alignment approximating the north bank of the channel to prevent further northward recession of the shoreline. Construction of this revetment would be begun in FY 1999 using O&M funding.

As the physical and numerical modeling efforts evolved, it was decided by representatives from the Jacksonville District, CHL, and Taylor Engineering, Inc. that investigation of the engineered channel alternative would be dropped from the physical modeling effort and would be simulated using the numerical model only. This decision was reached based on limitations associated with the non-tidal capabilities and the proximity of the engineered channel to the northern boundary of the physical model. It was felt to be more beneficial to spend more time testing the south jetty extensions and north jetty weir openings. Preliminary numerical simulations indicated that the engineered channel alternative provided many adverse impacts to shoaling in the inlet. Also, natural erosion of the sand spit was proceeding at such a rate that the shoreline would be at or near the proposed channel alignment by the time channel construction could begin, thus making the channel excavation an unnecessary expense.

The numerical model simulations were run with the 1000-foot parallel south jetty extension in place, and no weir opening in the north jetty. The initial channel cut was 2500 feet long, extending through the sand spit in the location of the southernmost relic channel. This configuration would extend the entrance channel westward along the alignment of the north jetty. The channel was 12 feet deep (mlw), 200 feet wide along the bottom, and 400 feet wide along the top, with 1 vertical: 8 horizontal side slopes. Numerical model simulations were made for initial, intermediate, and long-range conditions. An interactive process was followed whereby sediment transport inferences,

using numerical hydrodynamic modeling results, were used to help schematize or project anticipated bathymetric response (sediment erosion and deposition) until a pseudo-equilibrium condition was reached for each of these modeling runs. A complete discussion of each model simulation is presented in the supplemental report; results are summarized in this section.

Long-term simulations suggest that extensive shoaling will occur south and west of the existing sand spit. The main navigation channel gradually shifts from the natural channel south of the spit to the engineered channel north of the spit. The engineered channel will gradually widen, and the existing natural channel (south of the spit) will eventually shoal. It is currently believed that the island created by cutting the channel would remain emergent, but the possibility remains that waves and tidal flow may eventually erode the island to elevations below mean low water. Both conditions were examined in the numerical model.

Analyses of recent surveys (1986, 1990, and 1994) and aerial photographs indicate the sand spit is retreating northward at a rate of 70 to 80 feet per year. Based on the continuation of this recession rate, the shoreline could reach the position of the north bank of the engineered channel by the year 2002 or 2003. In the meantime, the entrance channel continues to straighten as the spit erodes, and erosional pressures on the north jetty are reduced. Construction of the engineered channel is not recommended at this time, since the spit is eroding naturally, and since construction of the channel could provide adverse shoaling and navigational impacts on the area. Construction of a revetment along the alignment of the north bank of the channel is recommended in order to prevent the northward recession of the shoreline past this point.

#### GENESIS SHORELINE MODELING.

The numerical shoreline change model GENESIS was used by Taylor Engineering, Inc to simulate the effects of the recommended plan on the adjacent shorelines north and south of the inlet. Separate simulations were conducted for the north and south beaches to determine the long-range (10 year) shoreline responses in each study area. As with the physical model, wave hindcast data from WIS stations 21 and 22 were used to define the input wave time series used to run the GENESIS model. The wave-transformation model RCPWAVE was used to transform waves from the WIS station locations to the nearshore pre-breaking reference line for each study area. Corps of Engineers and Florida Department of Environmental Protection (FDEP) surveys were used to develop the relative shoreline positions for model calibration, verification, and simulations.

The length of simulated shoreline for the south beach was 5.4 miles, with the northern boundary of the model located at the south jetty. GENESIS results for the south beach indicated that all three jetty extension cases (500, 800, and 1000 feet) produce increasing amounts of shoreline advance with increasing jetty extension lengths. Ten-year model simulations indicate shoreline advances of 130, 165, and 180 feet

immediately south of the south jetty, as compared to the no-project simulation. In all cases, the shoreline advance along the northern mile of the south beach occurs rapidly at the beginning of the simulation period, and quickly stabilizes. All three jetty extension lengths produce slight accretion along a reach of shoreline beginning about 1.3 miles south of the inlet and extending southward.

GENESIS results for the north beach were inconsistent with observed shoreline responses. In an effort to resolve this problem, the RCPWAVE model was discarded for north beach simulations, in favor of the internal GENESIS wave transformation model. Results improved dramatically and the internal model was used for all north beach modeling. Ten-year simulations indicated accretion along the entire 4.9-mile length of the north beach GENESIS grid. The amount of shoreline advance decreases with distance from the north jetty. The effects of lengthening the south jetty on the north beach could not be modeled; these model results reflect only long-term behavior without the effects of any jetty extensions, weir openings, or beach fills.

## PHYSICAL & NUMERICAL MODELING SUMMARY & CONCLUSIONS

The recommended plan of improvement at Ponce de Leon Inlet consists of a 1000-foot extension of the south jetty aligned parallel to the north jetty. A revetment extending landward from the western end of the north jetty would be constructed under separate O&M authority to halt the northward recession of the sand spit. The recommended plan is shown in figure 3. The south jetty extension was selected based on dye and tracer studies. The numerical model verified the improved flow characteristics of this alternative, and suggested that the parallel configuration would provide more favorable hydraulic responses in the inlet than the straight extension, primarily by providing less constriction of the tidal flow.

Re-opening the weir in the north jetty was not recommended. Flow through the weir was found to be minimal and insufficient to reduce scour along the north jetty. The increased depths along the north jetty allowed a high percentage of flow to bypass the weir influence and continue out through the mouth of the inlet. Current velocities remain excessive for sediment deposition with each of the three weir options in place, and construction of the proposed scour apron (described later in this appendix) will prevent future scour damage to the north jetty. Additionally, sediment passing through the weir would be transported throughout the inlet by strong ebb and flood currents, and increased channel shoaling would be very likely. Wave heights are also increased by re-opening the weir, as are cross-currents which would tend to pull boats through the weir. The increased channel shoaling and hazards to navigation outweigh any benefits to re-opening the weir.

Constructing the channel through the north spit was also not recommended. Extensive shoaling in the channels south and west of the spit would provide hazards to navigation. As the spit erodes naturally, the entrance channel will become more aligned with the axis of the inlet throat, tidal flow will become more evenly distributed across the



Ponce de Leon Inlet	
Reclamation Study	
Scale	1" = 100'
North Arrow	North
Figure 3	

inlet cross-section, and erosion around the base of the north jetty will decrease. Construction of the engineered channel is therefore regarded as an unnecessary expense, and is not recommended. However, construction of a revetment along the alignment of the north side of the proposed channel is recommended and will be provided under the O&M program in order to stop the northward recession of the sand spit shoreline. Construction of this revetment will allow the entrance channel to realign along the axis of the inlet while protecting the north jetty from flanking during storms. The revetment will also protect the wetland habitat along the northern portion of the sand spit from erosion, and will deflect the strong tidal currents away from the marinas located to the northwest of the structure.

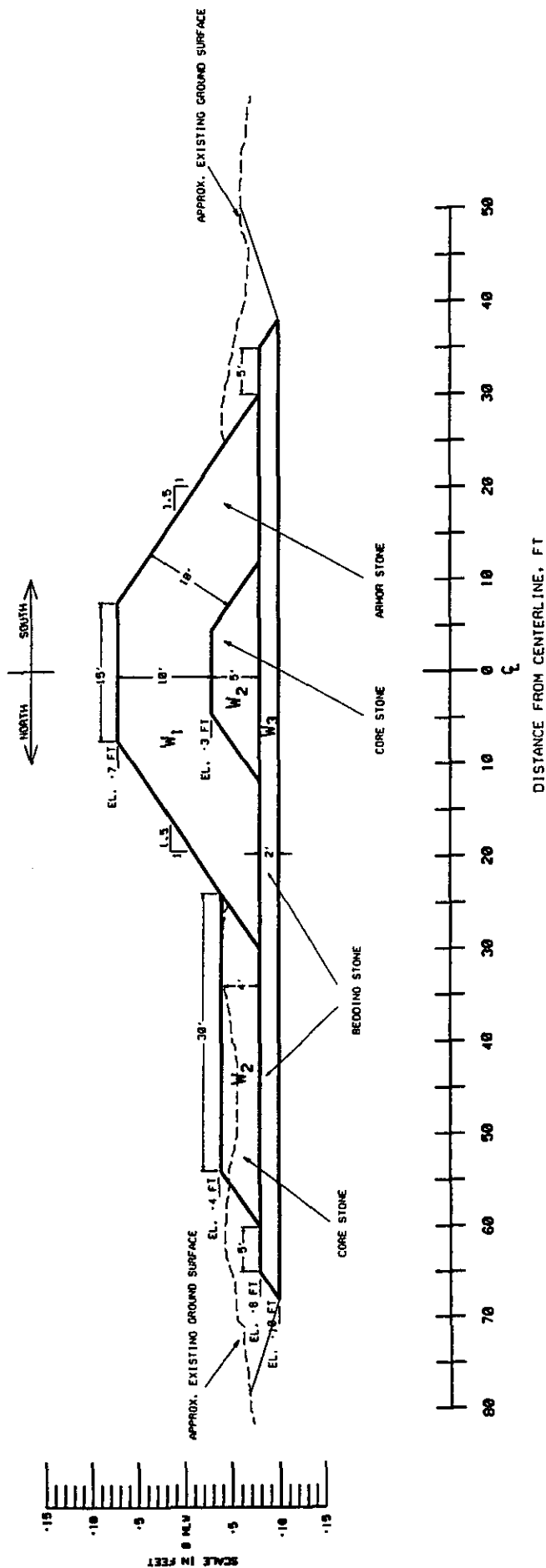
## DETAILED DESIGN OF RECOMMENDED PLAN

### DESIGN OF SOUTH JETTY EXTENSION

Construction of a 1000-foot seaward extension (parallel alignment) of the south jetty is recommended in order to reduce the northward transport of material into the inlet. Construction of the jetty extension will also distribute tidal currents more evenly across the inlet throat, causing a gradual deepening of the inlet throat and pulling the existing natural channel away from the north jetty, and more towards the center of the inlet. A plan view of the jetty extension is shown in figure 3; a cross-section is shown in figure 4.

The cross-section of the 1000-foot extension is similar to the cross-section used during the original jetty construction. The only modifications made were a steepening of the side slopes from 1:2 to 1:1.5, and the addition of a 30-foot scour apron on the inside (north side) of the jetty, to prevent damage to the jetty from the scouring which is expected upon completion of the extension. The crest elevation of the extension will match the elevation of the original jetty, but the crest width will increase from the jetty's 10 feet to 15 feet along the extension, in accordance with current design procedures which require a minimum of 3 stones across the width of the structure. A taper will be constructed to provide a smooth transition from the original jetty's 10-foot width to the extension's 15-foot width. Lower jetty crest elevations were examined as a cost-saving measure, but due to the relatively low crest elevation of the jetty extension (+7.0 feet mlw) coupled with the relatively high tide range (4.7 feet along the open coastline in South Daytona), excessive wave overtopping occurs for lower crest elevations during the higher tide levels. This in turn results in excessive wave heights inside the inlet, and the potential to transport excessive amounts of sediment over and through the jetty. It was therefore determined that +7 feet mlw was the minimum acceptable crest elevation for the jetty extension.

Armor stone size will range from 8 to 12 tons, with 50 percent of the stones weighing 10 tons or more. All armor stone will be 165 pcf. The armor layer will be 10 feet (2 stones) thick and the crest of the jetty will be 15 feet (3 stones) wide. The armor stone will overlie the intermediate and bedding layers as shown in figure 4. The intermediate layer is comprised of 500 to 2500 - lb stone, with 50 percent of the stones



#### NOTES:

1. STONE SIZES AS FOLLOWS:  
 $W_1$  : MEDIAN STONE SIZE - 10 TONS  
 $W_2$  : CORE STONE, 500 - 2500 LBS.  
 $W_3$  : BEDDING STONE, 1 - 12".
2. UNIT WEIGHT OF ARMOR AND CORE STONE TO BE NLT 165 PCF.  
UNIT WEIGHT OF BEDDING STONE TO BE NLT 140 PCF.
3. MATERIAL REMOVED DURING EXCAVATION IS TO BE PLACED SOUTH OF CONSTRUCTION AREA.

### PONCE DE LEON INLET FEASIBILITY STUDY

1000-FT SOUTH JETTY EXTENSION  
CROSS-SECTION VIEW  
FIGURE 4

weighing 1500 lbs or more. This stone will be used for construction of the core and scour apron, as shown in figure 4. The scour apron will be 4 feet thick, and the thickness of the core will vary according to depths along the jetty alignment. The 2-foot thick bedding layer will be constructed using standard Department of Transportation gradations for limerock. This stone has a unit weight of 140 pcf, and is available locally. Total quantities of materials required for construction of the 1000-foot south jetty extension are: 32,740 tons of 10-ton armor stone, 12,856 tons of 1500-lb core stone, 10,307 tons of bedding stone, 11,780 square yards of filter fabric, and 25,000 cubic yards of excavation.

The elevation of the foundation of the jetty will be at -10 feet mhw or on existing bottom, whichever is deeper. Any excavated material will be placed to the south of the jetty, outside of the footprint of construction. All excavated material is expected to be beach-quality sand.

## MAINTENANCE OPERATIONS

General. Construction of the Ponce de Leon Inlet jetties began in 1968 and was completed in 1970. Since that time some areas of localized scouring and foundation settlement have occurred along portions of the north and south jetties. Two areas are of particular interest: a large scour hole which threatens to undermine a 400-foot length of the landward end of the north jetty, and low areas along the crest of the north jetty. Both of these areas will be repaired using Operations and Maintenance (O & M) funds, and are mentioned here in the interest of providing a complete description of all ongoing Federal work at Ponce de Leon Inlet. Construction of the landward extension of the north jetty will also be an O & M function, as described previously. The design of these O&M structures will be discussed briefly in this section.

North Jetty Scour Hole. Due to the dogleg configuration of the inlet throat created by the presence of the sand spit, the ebb tidal flow through the inlet impinges on the landward section of the north jetty. Prior to the closure of the weir in 1984, a large shoal created by the transport of sediment through the weir protected this portion of the north jetty from scour damage. Following closure of the weir, the shoal rapidly eroded and the tidal scouring began to undermine the south side of the north jetty along the landward portion of the structure. The most critical area is located along a 400-foot section of the structure between stations 45+50 and 49+50, or along the seaward 400 feet of the north jetty concrete walkway. Construction plans show that the existing bottom along the jetty alignments was generally less than 10 feet deep at the time of jetty construction. Soundings taken in June 1996 indicated depths in excess of 45 feet along most of the length of the scour hole.

In order to better determine the condition of the jetty's foundation, a diver survey was performed in June/July 1996. This survey provided cross-sections at 100-foot intervals along a 1400-foot length of the north jetty. This survey area was chosen because the previous inlet survey had indicated depths in excess of 30 feet along this portion of the structure. As a result of the diver survey, it was determined that the most

critical area lay along a 400-foot reach between stations 45+50 and 49+50. Areas seaward of station 45+50 are still adequately protected by the scour apron which was constructed in 1979. Areas landward of station 49+50 have not experienced severe scouring at this time; however the area will be protected against potential future damage by the placement of a 40-foot wide scour apron, which will tie into the proposed (westward jetty extension) revetment at station 58+00.

The proposed renovation of the north jetty due to scour damage consists of rebuilding the side slope along the affected 400-foot reach, and the addition of a scour apron extending from the landward limit of the 1979 scour apron at station 45+50 to the location of the revetment tie-in at station 58+00. Armor stone will be placed at a 1 vertical:1 horizontal slope along the 400-foot reach, in order to stabilize the jetty side slope. Armor stone will have a median stone size of 10 tons, and will match the stone used to construct the jetty. The scour apron will be 3 feet thick and 40 feet wide, measured from the toe of the armor stone. Scour apron stone will be graded from 500-2500 lbs. A 2-foot thick layer of bedding stone will be placed under the armor and scour protection stones. A total of 9240 tons of armor stone, 9380 tons of intermediate stone, and 5710 tons of bedding stone will be required.

The sand spit (located south of the north jetty) is currently eroding at an average rate of about 70 feet per year. As the sand spit continues to erode the dogleg in the channel will continue to straighten, and the erosional pressure on the north jetty should decrease. It is anticipated that the sand spit shoreline will erode northward to the position of the proposed revetment by the year 2002 or 2003. Based on the performance of the 1979 scour apron, the proposed north jetty rehabilitation/scour apron should provide adequate protection of the north jetty during this time.

Low Areas Along North Jetty Crest. A centerline survey conducted in 1996 indicated that the entire length of the north jetty seaward of the eastern end of the concrete walkway has settled and average of 2 to 3 feet. Two areas in particular have settled approximately 7 to 8 feet, and are currently at or below mlw. A gap approximately 50 feet wide is centered at station 35+60, and a gap approximately 25 feet wide is centered at station 31+60. Significant wave energy has been observed passing through these gaps during storms, creating a potential hazard to navigation and allowing excessive sediment to enter the inlet.

Repair of these low areas will consist of the replacement of 10-ton armor stones across the two gaps. No attempt will be made to rebuild the jetty to the original +7.0-foot mlw elevation at this time; stone placement will blend smoothly into the adjacent crest elevations and widths (typically +5 feet elevation, 8-10 feet wide). Repairs of these low areas could be performed under the same Operations and Maintenance contract as the construction of the scour apron described above, and the same size armor stone will be used in both areas.

Revetment Construction / Westward Extension of the North Jetty. The sand spit inside the inlet has eroded consistently since weir closure in 1984. The current erosion rate of this shoreline is approximately 70-80 feet per year, and at this rate by the year 2002 or 2003, the sand spit shoreline will have eroded northward to the north jetty alignment. The primary purpose of the revetment is to prevent erosion in the vicinity of the western end of the north jetty, which could eventually result in flanking and failure of the jetty. The revetment would also prevent the shoreline from eroding through the peninsula of land shown in figure 3. Erosion of this peninsula would result in direct impact of strong tidal currents on the marinas located directly northwest of the peninsula, and loss of valuable wetland habitat.

The proposed revetment alignment is shown in figure 3. As shown this figure, the revetment alignment is curved, extending first in a southwesterly direction (jutting into the inlet) from the landward end of the north jetty, then turning to more of a westerly direction which is more aligned with the axis of the inlet throat. This alignment was selected in order to protect existing structures in the adjacent public park, to follow the natural alignment of the shoreline of the peninsula (behind the spit) as shown in figure 3, to provide a maximum degree of protection to the adjacent marinas, and to protect a maximum amount of wetland habitat. Due to the varying exposure to ocean waves along the length of the structure, the revetment will be constructed using two cross-sections, and could be constructed in stages if necessary. The first section (revetment section #1) is 800 feet in length, and extends from the intersection of the north jetty southwestward to the point where the alignment bends to the west. The second section (revetment section #2) begins at this point, and extends an additional 1540 feet westward, along the southern shoreline of the peninsula shown in figure 3.

According to a recession analysis conducted by Taylor Engineering Inc. (contained in the physical and numerical modeling Supplemental Report), the present rate of northward recession of the sand spit shoreline is about 70 feet per year. At this rate, the widest portion of the sand spit should recede to the position of the revetment alignment by the year 2002 or 2003. Due to the more eroded condition of the shoreline along revetment section #1 and the more immediate concern of preventing flanking of the north jetty, revetment section #1 will be constructed first, in FY 1999. Revetment section #2 would be constructed 3-4 years later, based on the present rate of shoreline erosion along the revetment alignment.

A brief summary of the design of the revetment is presented in this section of the appendix (even though it will be constructed with O & M funding and is not a part of the plan recommended herein), because of the eventual impact of the revetment on the stability of the inlet once the sand spit shoreline recedes to the revetment alignment. Details of the design of each revetment section are presented below.

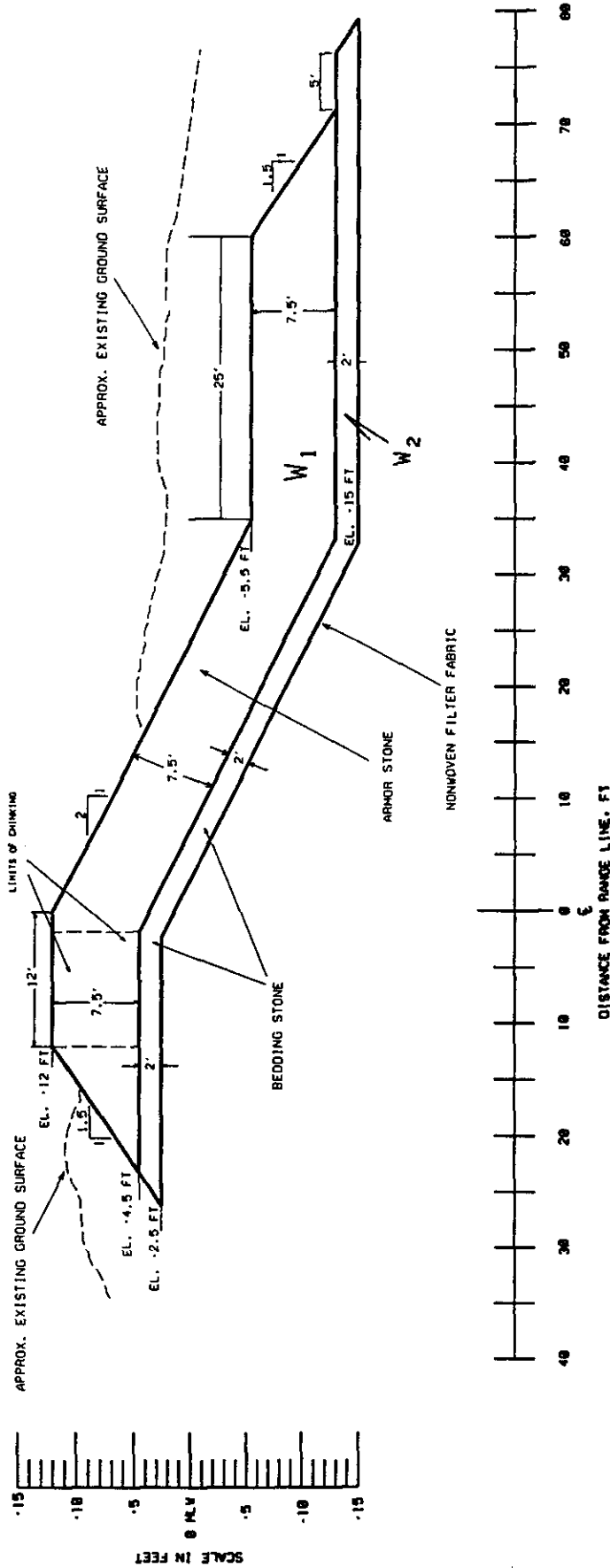
Design of Revetment Section #1. This landward extension of the north jetty is referred to as a revetment because the cross-section of the structure is in the form of a revetment, not the symmetrical cross-section of a jetty. Both terms "revetment" and

“landward extension of the north jetty” are used interchangeably throughout this design appendix.

Revetment section #1 will be more directly exposed to ocean waves than revetment section #2, and will be constructed of 8500-lb armor stone. This armor stone is designed to withstand 8-foot, 12-second waves. Under the present inlet configuration it is very unlikely that 8-foot waves could reach the revetment due to the shallow shoals through the inlet throat, but following construction of the south jetty extension, depths throughout the inlet throat are expected to gradually increase, and the design wave would then be able to reach the structure. The 8-foot design wave represents a maximum probable wave height which could impact directly on the side slope of the revetment. Assuming that deep water existed from the toe of the revetment to the open ocean (following erosion of the inlet shoals) the design wave could impact the first section of the structure when water depths at the toe of the revetment are 10.2 feet deep. The most critical wave breaking condition will occur over the scour apron (which is 5.5 feet deep), with the addition of the 2.7-foot spring high tide and a 2-foot storm surge.

A cross-section of this section of the revetment is shown in figure 5. The revetment ties into the existing north jetty at station 58+00. The revetment crest elevation is +12 feet, mhw, which matches the crest elevation of the north jetty and the existing topography behind this portion of the revetment alignment. A front slope of 1 vertical on 2 horizontal will be constructed, and a 25-foot scour apron will be constructed at the toe of the revetment to allow for future scouring to depths of 30 feet, which are currently observed in this region of the inlet. A 2-foot thick layer of bedding stone will be placed over nonwoven geotextile filter fabric as shown in figure 5. Bedding stone will be standard Department of Transportation graded Florida limerock, with a density of 140 pcf. Armor and intermediate stone will be 165 pcf. The crest of the revetment will be chinked with bedding stone from elevation +4.5 to +12, in order to decrease the permeability of the revetment and minimize loss of backbeach material through the structure. The approximate limits of chinking are shown in figure 5. Total quantities of materials required for the construction of revetment section #1 are: 35,692 tons of 8500-lb armor stone, 8,538 tons of bedding stone, 768 tons of chinking stone (same gradation as bedding stone), 9780 square yards of filter fabric, and 60,000 cubic yards of excavation.

Design of Revetment Section #2. The second revetment section is located further into the inlet, and is oriented almost parallel to the axis of the inlet throat. Ocean waves will impact this section of the revetment much more obliquely than section 1, and wave impact forces will be reduced accordingly. The 2500-lb stone size is adequate to withstand direct impacts from 5-foot, 12-second waves. Large ocean waves are rarely observed this far inside the inlet under the current inlet configuration, but boat wakes in excess of 2 to 3 feet are observed on a continuous basis.



#### NOTES:

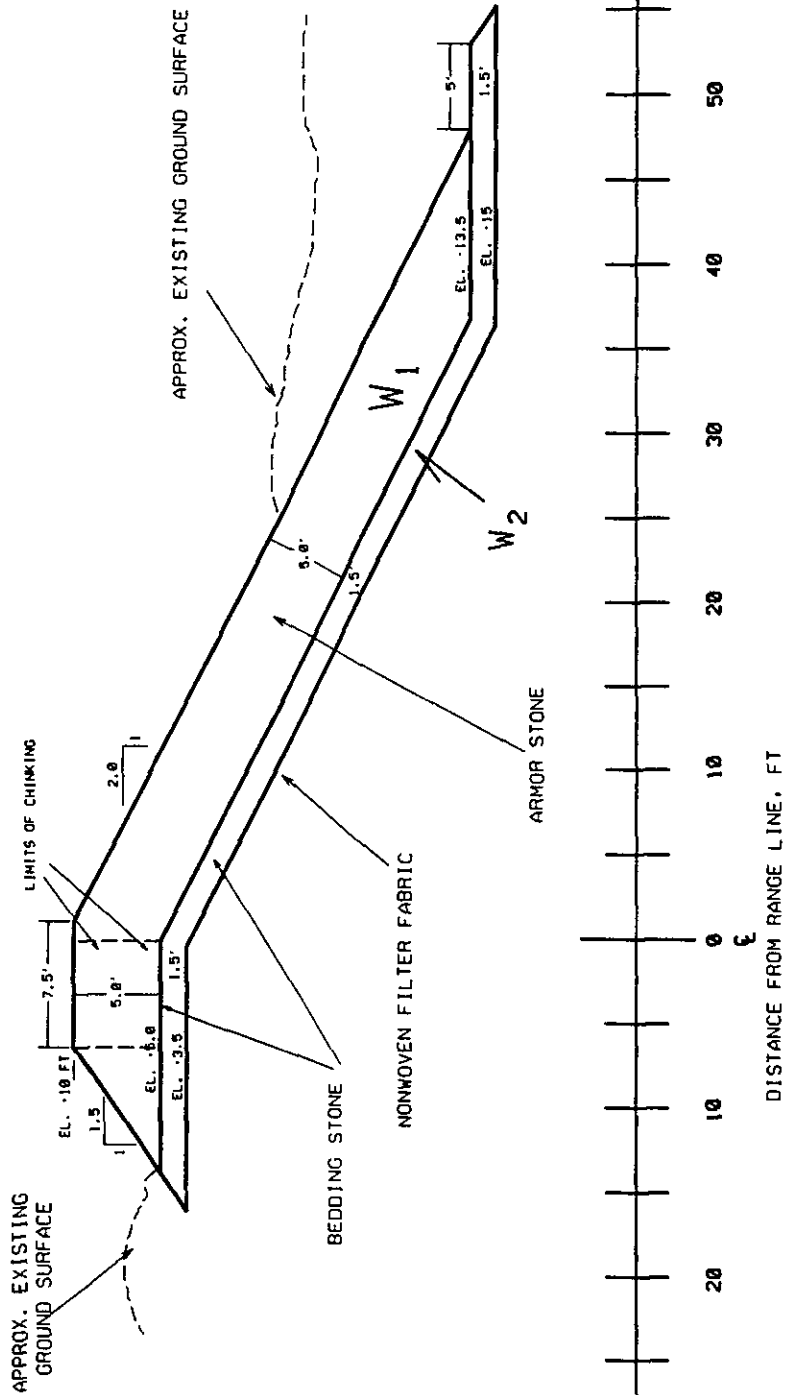
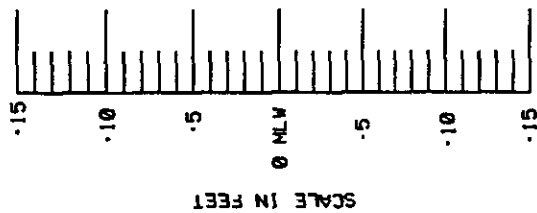
1. STONE SIZES AS FOLLOWS:  
 $W_1$  : MEDIAN STONE SIZE - 8500 LBS.  
 $W_2$  : BEDDING STONE: 2 - 90 LBS.
2. FILTER FABRIC (NONWOVEN) WILL BE PLACED UNDER BEDDING LAYER
3. UNIT WEIGHT OF ARMOR STONE TO BE NOT 165 PCF. UNIT  
 WEIGHT OF BEDDING STONE TO BE NO LESS THAN 140 PCF.
4. MATERIAL REMOVED DURING EXCAVATION IS TO BE  
 REPLACED OVER TOE OF RETENTION, TO APPROXIMATE  
 PRE-CONSTRUCTION GRADE. ANY ADDITIONAL MATERIAL  
 WILL BE PLACED AS BACKFILL.

PONCE DE LEON INLET  
 FEASIBILITY STUDY  
 REVETMENT SECTION • 1  
 TYPICAL CROSS SECTION  
 FIGURE 5

A cross-section of revetment section #2 is shown in figure 6. A lower crest elevation is used along section #2 due to lower wave heights and the lack of potential damage which would be caused by overtopping. The base of the structure extends down to elevation -15, mhw. A lower degree of scouring is expected in this region of the inlet, and no scour apron is provided. Other differences between revetment sections 1 and 2 include a 1-foot higher base elevation of the crest of the structure (+3.5 feet, mhw) in order to achieve the design crest elevation of +10 feet with a 2-stone thick armor layer. A 1.5-foot thick bedding layer will be required instead of the 2-foot layer used in section #1. The same nonwoven geotextile filter fabric and bedding stone gradations will be used along both sections of the revetment. The crest of section #2 will be chinked in the same manner described above for section #1.

All stone sizes and gradations for the revetment were calculated using the Automated Coastal Engineering System (ACES v.1.07 software developed by the Coastal Engineering Research Center. As per Shore Protection Manual criteria, the armor layer is at least 2 stones thick, and the crest of the structure is 3 stones wide. This results in an armor layer 7.5 feet thick and a crest 12 feet wide for section #1, and an armor layer 5.0 feet thick and crest 7.5 feet wide for revetment section #2. Table 1 provides the gradations of each armor and bedding layer. No intermediate layer is required due to the wide gradations specified for the armor and bedding layers. Incident waves with a 12-second period produce the maximum required stone sizes for both revetment sections. The 12-second wave period is not uncommon for this area and was therefore used for all designs. Other ACES input included a 1:50 nearshore slope, a 1:2 revetment front slope, 10-foot depth at toe of structure, armor stone unit weight of 165 pcf, permeability coefficient of 0.1, and damage level of 2%.

Armor and bedding stone may be transported to the site by truck or barge, at the contractor's discretion. Excavated material will be placed behind (north of) the revetment, and will be used primarily to transition from the revetment crest to existing natural grade. Excavated backfill will also be placed in front of the revetment for aesthetic purposes. Temporary stockpile areas will be provided immediately north of the revetment, and along the beach north and south of the landward end of the north jetty, seaward of the dunes. Total quantities of materials required for construction are: 30,056 tons of 2500-lb armor stone, 8,476 tons of bedding stone, 610 tons of chinking stone, 13,000 square yards of filter fabric, and 40,000 cubic yards of excavation.



NOTES:

1. STONE SIZES AS FOLLOWS:  
W<sub>1</sub> : MEDIAN STONE SIZE - 2500 LBS  
W<sub>2</sub> : BEDDING STONE - 1-25 LBS
2. NONWOVEN FILTER FABRIC WILL BE PLACED UNDER BEDDING LAYER
3. MATERIAL REMOVED DURING EXCAVATION IS TO BE REPLACED OVER TOE OF REVETMENT. TO APPROXIMATE PRE-CONSTRUCTION GRADE. ANY ADDITIONAL MATERIAL WILL BE PLACED AS BACKFILL.

PONCE DE LEON INLET  
FEASIBILITY STUDY  
REVETMENT SECTION #2  
TYPICAL CROSS SECTION  
FIGURE 6

**TABLE 1**  
**STONE SIZE GRADATIONS**  
**REVTMENT SECTION #1**

ARMOR STONE (165 pcf)		
% Less than by weight	Weight (lbs)	Dimension (ft)
0	1100	1.9
15	3500	2.8
50	8500	3.8
85	17000	4.7
100	34800	5.9

BEDDING STONE (140 pcf)		Sieve Size
% Passing		
100		12"
65-85		9"
25-45		3"
0-10		1"

**STONE SIZE GRADATIONS**  
**REVTMENT SECTION #2**

ARMOR STONE (165 pcf)		
% Less than by weight	Weight (lbs)	Dimension (ft)
0	310	1.2
15	1000	1.8
50	2500	2.5
85	5000	3.1
100	10000	3.9

BEDDING STONE (140 pcf)		Sieve Size
Percent Passing		
100		12"
65-85		9"
25-45		3"
0-10		1"

## GEOTECHNICAL INVESTIGATION

### 1. Geotechnical Investigations

The subsurface investigation for this project consist of drilling 3 core borings along the proposed channel excavation alignment, drilling 4 core borings for the proposed 1540-foot revetment on the north shore and drilling 2 core boring for the 1000 foot jetty extension. The core borings drilled for the channel excavation and north shoreline revetment was drilled to depths up to 30 feet deep. The two core borings drilled for the proposed jetty extension, CB-PDLJ-2 and CB-PDLJ-3 was drilled to depths of 36 and 55 feet respectfully. See figure 7 for core boring layout.

### 2. Material Encountered

The material encountered in the seven core borings drilled for the proposed channel excavation and shore line revetment consisted of medium to dense, slightly silty quartz sand with fine to Coarse grained shell fragments (SP AND SP-SM material).

The material encountered in the two core borings drilled for the proposed jetty extension consisted of 36 foot of medium to dense, poorly graded sand (SP) with some of shell fragments. Below the sand layer boring CB-PDJ-2, was a soft to medium dense layer of highly plastic clay. The 39 feet of medium to dense sands above this clay zone will be of sufficient to support the proposed structure.

### 3. Slope Stability

The channel excavated slopes were analysis for end of construction case. The analysis indicated that the excavated slope of 1 vertical on 2.5 horizon would be stability.

## COST ESTIMATES

Cost estimates for the recommended plan are contained in enclosure 2, at the end of this appendix. Cost estimates are provided in MCASES format, and are referenced to October 1996 price levels.

